GEOTECH
CONSULTANTS, INC.

APR 1 0 2007

13256 Northeast 20th Street, Suite 16
Bellevue, Washington 98005
(425) 747-5618 FAX (425) 747-8561

April 9, 2007

JN 05402

Ariel Development
3317 Third Avenue South
Seattle, Washington 98134

Attention: Tim Farrell

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Residence Remodel and Additions
3100 Airport Way South
Seattle, Washington

via email tim@arieldevelopment.com

Dear Mr. Farrell:

We are pleased to present this geotechnical engineering report for the proposed remodel of the existing building in Seattle, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design criteria for foundations and retaining walls. This work was authorized by your acceptance of our proposal, , dated, 2005.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

D. Robert Ward, P.E.
Principal

DRW: jyb

GEOTECHNICAL ENGINEERING STUDY
Remodel to the Existing Building
3100 Airport Way
Seattle, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed remodel and additions to the residence in Seattle.

Based on discussions that we have had with the project design team, we understand that the lower floor of the building will be lowered approximately 4 feet. The existing upper floor will remain. A new middle floor will be placed over the new lower floor.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the Rainier Commons complex (formerly the Rainier Brewery site) in the SoDo area of Seattle. The building that will be remodeled is located northwestern portion of the Rainier Commons complex, situated about 80 feet east of Airport way. We understand that the building is likely over 100 years old. A relatively flat, paved parking lot is located between the street and subject building. The lot rises about three feet adjacent to the western edge of the building. The building's slab floor, which is now removed, had a grade that was approximately 7 feet above parking lot. The ground on the eastern side of the building, which is nearly flat and covered with asphalt, has a grade that is about 4 to 5 feet above the slab floor. Other buildings of the complex are located to the east. They generally step up the hillside that rises to the east.

The building has approximately 25 feet of inside, vertical clearance on its southern end. An upper level exists on the northern end of the building, and the clearance in that area is about 15 feet. A stairway at the northeastern corner of the building provides access to the upper level. We observed several small cracks in the brick structure of the building. The most significant crack is a vertical one that is located on the eastern wall of the building.

It is apparent that the building was constructed with brick and has a concrete foundation. Based on test holes that were excavated inside and outside of the building, we found that the depth of most of continuous perimeter footings between perimeter columns are just below the outside grade on the eastern side of the building and just below the inside of the building on the eastern side. It appears that column footings are slightly deeper.

SUBSURFACE

The subsurface conditions in the building area were explored by drilling one test boring near the northwestern corner of the building as shown on the Site Exploration Plan, Plate 2. We also explored the soil in the side of the building using hand equipment. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The borings were drilled on November 2, 2005 using a trailer-mounted, hollow-stem auger drill. Samples were taken at 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test boring, and obtained representative samples of the soil encountered. The Test Boring Log is attached as Plate 3.

Soil Conditions

The test boring encountered approximately 12 feet of loose and soft sandy silty and silt near the ground surface. This soil was underlain by approximately of loose sand that became medium-dense at a depth of approximately 17 feet. This sand was underlain by medium-stiff to stiff clayey silt to the maximum explored depth of 39 feet. We used a 1/2-inch steel bar to probe the ground adjacent to the eastern side of the building below the slab level. We found medium-dense soil at a depth of approximately 7 feet.

We observed the drilling of a test boring for a separate project near the very northwestern corner of the Rainier Commons complex, about 30 feet from Airport Way; approximately XX feet of the upper loose soil was revealed there. It is known that the SoDo district is an old tide flat where the upper soils are loose for depths greater than 30 feet. We also excavated test pits on the hillside on the eastern side of the complex; more stiff silt was found near the ground surface there. It is apparent that the transition between the looser, old tide flat soils and the stiffer hillside soils occurs near the eastern side of the subject building.

No obstructions were revealed by our explorations. However, debris, buried utilities, and old foundation and slab elements are commonly encountered on sites that have had previous development.

Groundwater Conditions

Groundwater seepage was observed in the bottom of the sand layer at a depth of approximately 19 feet. However, the test boring was left open for only a short time period. Therefore, the seepage levels on the logs represent the location of transient water seepage and may not indicate the static groundwater level. Groundwater levels encountered during drilling can be deceptive, because seepage into the boring can be blocked or slowed by the auger itself.

It should be noted that groundwater levels vary seasonally with rainfall and other factors. We anticipate that groundwater could be found higher in the sand layer later in the normally wet winter and spring months.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. Where a transition in soil type occurred between samples in the borings, the depth of the transition was interpreted. The relative densities and moisture descriptions indicated on the test boring logs are interpretive descriptions based on the conditions observed during drilling.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The explorations conducted for this study encountered soft/loose soils to a depth of about 15 feet below the new proposed slab on the western side of the building, and approximately 4 feet below the eastern side of the building. We understand from meetings with the design team that all new building loads will be supported on new foundations. This is prudent because it appears that the existing foundations bear on soft/loose soils and are thus in a marginal condition; additional loads to the existing foundation would likely cause future settlement. We believe that there are two options for new foundations: a deep foundation system consisting of driven steel pipe piles (this type of pile system can be done readily from within the existing building, or lightly-loaded footings. The pipe pile system would provide a system where future settlement would be negligible and deter the possibility for seismic liquefaction. The footing system may be more economical, but there would be a possibility of settlement as noted in the subsequent sections of this report.

Excavations in the site soils should not be made steeper than 1:1 (Horizontal:Vertical). New foundations on the inside of the eastern side of the building should also be positioned so that they are outside of an imaginary 1:1 (H:V) line that extends below the existing eastern footing. If new walls are placed inside of this imaginary line, mechanical lateral restraints would first have to be incorporated into the existing foundation prior to the construction of the new foundation. As noted earlier, the continuous footings along the eastern side of the building are buried just below the original slab grade.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking and bathing. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The designer or architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a build up of excessive water vapor within the planned structure.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

We recommend including this report, in its entirety, in the project contract documents. This report should also be provided to any future property owners so they will be aware of our findings and recommendations.

SEISMIC CONSIDERATIONS

In accordance with Table 1615.1.1 of the 2003 International Building Code (IBC), the site soil profile within 100 feet of the ground surface is best represented by Soil Profile Type D (Stiff Soil). As required by the Critical Areas Ordinance, the design criteria presented in this report consider the effects of a one-in-100-years seismic event. The sandy site soils found in the test boring between approximately 12 and 17 feet are moderately susceptible to seismic liquefaction is a large seismic event occurs when the sand is saturated.

Seismic Liquefaction

The building is underlain by loose, possibly-saturated soil consisting of silty sand, sand, and sandy silt. Soils such as those found in the test boring have been demonstrated to have a moderate potential for liquefaction during a large earthquake. Current geotechnical analysis cannot accurately predict where and to what extent soil liquefaction will occur during a large earthquake. It is therefore prudent to assume that soil liquefaction could occur beneath the site. The study of liquefaction and its resulting effects is ongoing, as development in areas underlain by saturated alluvium or hydraulic fill has only really occurred to a great extent in the last 20 to 30 years. Recent observations from earthquakes occurring in the State of California and in Japan indicate the following information about structures in areas underlain by liquefiable soil:

- Ground surface subsidence due to liquefaction tends to occur either over a large area or at concentrated points where sand boils occur.
- Differential foundation settlement typically occurs either at the location of a sand boil or where the subsurface soil conditions change significantly.
- Catastrophic foundation settlement due to liquefaction occurs primarily as a result of lateral spreading, particularly in waterfront areas.
- Conventionally constructed commercial buildings have not been documented to exhibit a high percentage of catastrophic foundation failures in liquefiable areas.

Due to the uncertainties in predicting the potential effects of seismic liquefaction on commercial structures in the SoDo valley, we recommend that all foundations either be supported on pipe piles that embed into stiff, non-liquefiable soils, or on continuous, low-bearing capacity footings that are placed on at least 1 feet of structural fill. This will allow the footings to span across any areas of concentrated liquefaction (sand boils) if they occur. Considering the recommendations presented in this geotechnical report, it is our professional opinion that the differential foundation settlement that could be experienced by the structure during a large earthquake should be on the order of 1 to 3 inches in a distance of 100 feet if a footing foundation is used.

By preventing catastrophic settlement of the footing foundations, the safety of the occupants should be protected. This conforms with the intent of Section 1626.1 of the 1997 UBC, which requires that the design "safeguards against major structural failures and loss of life." The intent is not to prevent damage or ensure continued function of the structure after the design seismic event.

The Puget Sound region is very seismically active, with hundreds of small (Magnitude (M) of less than 3.0) earthquakes occurring every year. Within the last approximately 100 years, at least six earthquakes having a M>6.0 have been recorded in the Puget Sound basin. Of these, a 6.8M earthquake was centered in Nisqually in 2001, a M7.1 earthquake was centered in Olympia in 1949 and a M6.5 earthquake occurred in Seattle area in 1965. In 1872, a M7.4 earthquake shook north-central Washington. This is the largest earthquake that has occurred in recent history. Currently, seismologists and geologists are studying geologic evidence that indicates subduction zone earthquakes with magnitudes of up to 8 to 9 have occurred every 300 to 500 years. The last known subduction zone quake of this magnitude possibly affected the Puget Sound region approximately 300 years ago. Based on the available information, and the current studies, it appears reasonable to assume that an earthquake having a magnitude of up to 7.5 could occur every 50 to 100 years in the Puget Sound region. Due to the large number of known and unknown faults in the area, it appears very difficult to accurately predict where a sizable earthquake will occur.

CONVENTIONAL FOUNDATIONS

The proposed structure could be supported on continuous spread footings bearing on undisturbed, at least 12 inches of structural. See the section entitled ***General Earthwork and Structural Fill*** for recommendations regarding the placement and compaction of structural fill beneath structures. Adequate compaction of structural fill should be verified with frequent density testing during fill placement. Prior to placing structural fill beneath foundations, the excavation should be observed by the geotechnical engineer to document that adequate bearing soils have been exposed. We recommend that continuous spread footings have minimum width of 16 inches. The footings should be designed to span at least 10 feet. This is so that settlement were to occur due to liquefaction, the footings could very likely span over the settled area. Exterior footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface for protection against frost and erosion. The local building codes should be reviewed to determine if different footing widths or embedment depths are required.

An allowable bearing pressure of 1,500 pounds per square foot (psf) should be used for the design of the footings. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction, static settlement of footings will be about one inch, with differential settlements on the order of one inch in a distance of 75 feet along a continuous footing with a uniform load.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level structural fill. We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.40
Passive Earth Pressure	250 pcf

Where: (i) pcf is pounds per cubic foot, and (ii) passive earth pressure is computed using the equivalent fluid density.

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend maintaining a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate values.

PIPE PILES

Several different pipe piles could be used for this project. A 2-inch-diameter pipe pile driven with a 90-pound jackhammer to a final penetration rate of 1 inch or less for one minute of continuous driving may be assigned an allowable compressive load of 2 tons. Extra-strong steel pipe should be used. Three- or 4-inch-diameter pipe piles driven with a 650- or 800- or 1,100-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (650-pound hammer)	FINAL DRIVING RATE (800-pound hammer)	FINAL DRIVING RATE (1,100-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
3 inches	12 sec/inch	10 sec/inch	6 sec/inch	6 tons
4 inches	20 sec/inch	15 sec/inch	10 sec/inch	10 tons

Note: The refusal criteria indicated in the above table are valid only for pipe piles that are installed using a hydraulic impact hammer carried on leads that allow the hammer to sit on the top of the pile during driving. If the piles are installed by alternative methods, such as a vibratory hammer or a hammer that is hard-mounted to the installation machine, numerous load tests to 200 percent of the design capacity would be necessary to substantiate the allowable pile load. The appropriate number of load tests would need to be determined at the time the contractor and installation method are chosen. As a minimum, load tests on 20 percent of the piles is typical where alternative pile installation methods are used.

As a minimum, Schedule 40 pipe should be used. The site soils should not be highly corrosive. Considering this, it is our opinion that standard "black" pipe can be used, and corrosion protection, such as galvanizing, is not necessary for the pipe piles. We recommend a minimum pile length of 10 feet to achieve embedment into competent native soils.

Seattle Director's Rule 12-2001 contains several prescriptive requirements related to the use of pipe piles having a diameter of less than 10 inches. Under Director's Rule 12-2001, load tests are not required for 2-inch-diameter piles that are driven with a 90-pound jackhammer and are designed for an allowable 2-ton capacity. Under Director's Rule 12-2001, load tests are required on 3 percent of the installed piles, with a minimum of one pile load test, for piles larger than 2 inches. Additionally, full-time observation of the pile installation by the geotechnical engineer-of-record is required by Director's Rule 12-2001. The City of Seattle limits the length of 2-inch-diameter pipe piles to 30 feet. If pile lengths exceed the 30 feet, a code alternate or modification must be applied for.

Pile caps and grade beams should be used to transmit loads to the piles. Isolated pile caps should include a minimum of two piles to reduce the potential for eccentric loads being applied to the piles. Subsequent sections of pipe can be connected with slip or threaded couplers, or they can be welded together. If slip couplers are used, they should fit snugly into the pipe sections. This may require that shims be used or that beads of welding flux be applied to the outside of the coupler.

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. For this condition, the foundation must be either poured directly against relatively level, undisturbed soil or surrounded by level, structural fill. We recommend using a passive earth pressure of 250 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate passive value. Due to their small diameter, the lateral capacity of vertical pipe piles is relatively small. However, if lateral resistance in addition to passive soil resistance is required, we recommend driving battered piles in the same direction as the applied lateral load. The lateral capacity of a battered pile is equal to one-half of the lateral component of the allowable compressive load, with a maximum allowable lateral capacity of 500 pounds for 2-inch piles, and 1,000 pounds for 3- and 4-inch piles. The allowable vertical capacity of battered piles does not need to be reduced if the piles are battered steeper than 1:5 (Horizontal:Vertical).

PERMANENT FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

PARAMETER	VALUE
Active Earth Pressure *	40 pcf
Passive Earth Pressure	250 pcf
Coefficient of Friction	0.40
Soil Unit Weight	120 pcf

Where: (i) pcf is pounds per cubic foot, and (ii) active and passive earth pressures are computed using the equivalent fluid pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure.**

The values given above are to be used to design permanent foundation and retaining walls only. The passive pressure given is appropriate for the depth of level structural fill placed in front of a retaining or foundation wall only. The values for friction and passive resistance are ultimate values and do not include a safety factor. We recommend a safety factor of at least 1.5 for overturning and sliding, when using the above values to design the walls. Restrained wall soil parameters should be utilized for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density.

Wall Pressures Due to Seismic Forces

The City of Seattle Critical Areas regulations require that a dynamic analysis of the structure and retaining walls be conducted. To model the surcharge wall loads that could be imposed by the design earthquake, we recommend adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 8**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment. The wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. Onsite soils should not be used as free-draining backfill.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls to reduce the potential for surface water to percolate into the backfill. The section entitled ***General Earthwork and Structural Fill*** contains recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing, and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a build up of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact a specialty consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

The ***General***, ***Slabs-On-Grade***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

SLABS-ON-GRADE

The building floors can be constructed as slabs-on-grade atop at least 12 inches of structural fill. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill. Due to the soft condition of the site soils, we recommend that reinforcing, such as #4 rebar at 18-inch-centers be placed in the slab to deter potential cracking.

Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. All interior slabs-on-grade must be underlain by a capillary break or drainage layer consisting of a minimum 4-inch thickness of gravel or crushed rock that has a fines content (percent passing the No. 200 sieve) of less than 3 percent and a sand content (percent passing the No. 4 sieve) of no more than 10 percent. As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI also notes that vapor *retarders*, such as 6-mil plastic sheeting, are typically used. A vapor retarder is defined as a material with a permeance of less than 0.3 US perms per square foot (psf) per hour, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where plastic sheeting is used under slabs, joints should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection. If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.00 perms per square foot per hour when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

In the recent past, ACI (Section 4.1.5) recommended that a minimum of 4 inches of well-graded compactable granular material, such as a 5/8 inch minus crushed rock pavement base, should be placed over the vapor retarder or barrier for protection of the retarder or barrier and as a "blotter" to aid in the curing of the concrete slab. Sand was not recommended by ACI for this purpose. However, the use of material over the vapor retarder is controversial as noted in current ACI literature because of the potential that the protection/blotter material can become wet between the time of its placement and the installation of the slab. If the material is wet prior to slab placement, which is always possible in the Puget Sound area, it could cause vapor transmission to occur up through the slab in the future, essentially destroying the purpose of the vapor barrier/retarder. Therefore, if there is a potential that the protection/blotter material will become wet before the slab is installed, ACI now recommends that no protection/blotter material be used. However, ACI then recommends that, because there is a potential for slab cure due to the loss of the blotter material, joint spacing in the slab be reduced, a low shrinkage concrete mixture be used, and "other measures" (steel reinforcing, etc.) be used. ASTM E-1643-98 "Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs" generally agrees with the recent ACI literature.

We recommend that the contractor, the project materials engineer, and the owner discuss these issues and review recent ACI literature and ASTM E-1643 for installation guidelines and guidance on the use of the protection/blotter material. Our opinion is that with impervious surfaces that all means should be undertaken to reduce water vapor transmission.

The ***General***, ***Permanent Foundation and Retaining Walls***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

Isolation joints should be provided where the slabs intersect columns and walls. Control and expansion joints should also be used to control cracking from expansion and contraction. Saw cuts or preformed strip joints used to control shrinkage cracking should extend through the upper one-fourth of the slab. The spacing of control or expansion joints depends on the slab shape and the amount of steel placed in it. Reducing the water-to-cement ratio of the concrete and curing the concrete, by preventing the evaporation of free water until cement hydration occurs, will also reduce shrinkage cracking.

We recommend proof-rolling slab areas with a heavy truck or a large piece of construction equipment prior to slab construction. Any soft areas encountered during proof-rolling should be excavated and replaced with select, imported structural fill.

EXCAVATIONS AND SLOPES

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Temporary cuts to a depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type B. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut. Permanent slopes should be inclined no steeper than 2:1 (H:V).

The above recommended temporary slope inclination is based on the conditions exposed in our explorations, and on what has been successful at other sites with similar soil conditions. It is possible that variations in soil and groundwater conditions will require modifications to the inclination at which temporary slopes can stand. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. It is also important that surface water be directed away from temporary slope cuts. The cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger. These recommendations may need to be modified if the area near the potential cuts has been disturbed in the past by utility installation, or if settlement-sensitive utilities are located nearby.

DRAINAGE CONSIDERATIONS

Foundation drains should be used where the lower slab is below the outside grade or where outside grade does not slope downward from a building. Drains should also be placed at the base of all earth-retaining walls. These drains should be surrounded by at least 6 inches of 1-inch-minus, washed rock and then wrapped in non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the bottom of a slab floor or the level of a crawl space, and it should be sloped for drainage. All roof and surface water drains must be kept separate from the foundation drain system. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains. The City of Seattle typically requires that Schedule 40 PVC pipe be used beneath the interior of structures.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Also, an outlet drain is recommended for all crawl spaces to prevent a build up of any water that may bypass the footing drains.

Groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, behind permanent retaining or foundation walls, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction. The following table presents recommended relative compactions for structural fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath footings, slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

The on-site soils are not suitable for reuse as structural fill, due to their wet and silty condition. Structural fill placed below the footings and/or the slab consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the explorations are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples in test borings. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of Ariel Development and its representatives, for specific application to this project and site. Our recommendations and conclusions are based on observed site materials and engineering analyses. Our conclusions and recommendations are professional opinions derived in accordance with current standards of practice within the scope of our services and within budget and time constraints. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

In addition to reviewing the final plans, Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

During the construction phase, we will provide geotechnical observation and testing services when requested by you or your representatives. Please be aware that we can only document site work we actually observe. It is still the responsibility of your contractor or on-site construction team to verify that our recommendations are being followed, whether we are present at the site or not.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plate 3	Test Boring Log

We appreciate the opportunity to be of service on this project. If you have any questions, or if we may be of further service, please do not hesitate to contact us.

Respectfully submitted,

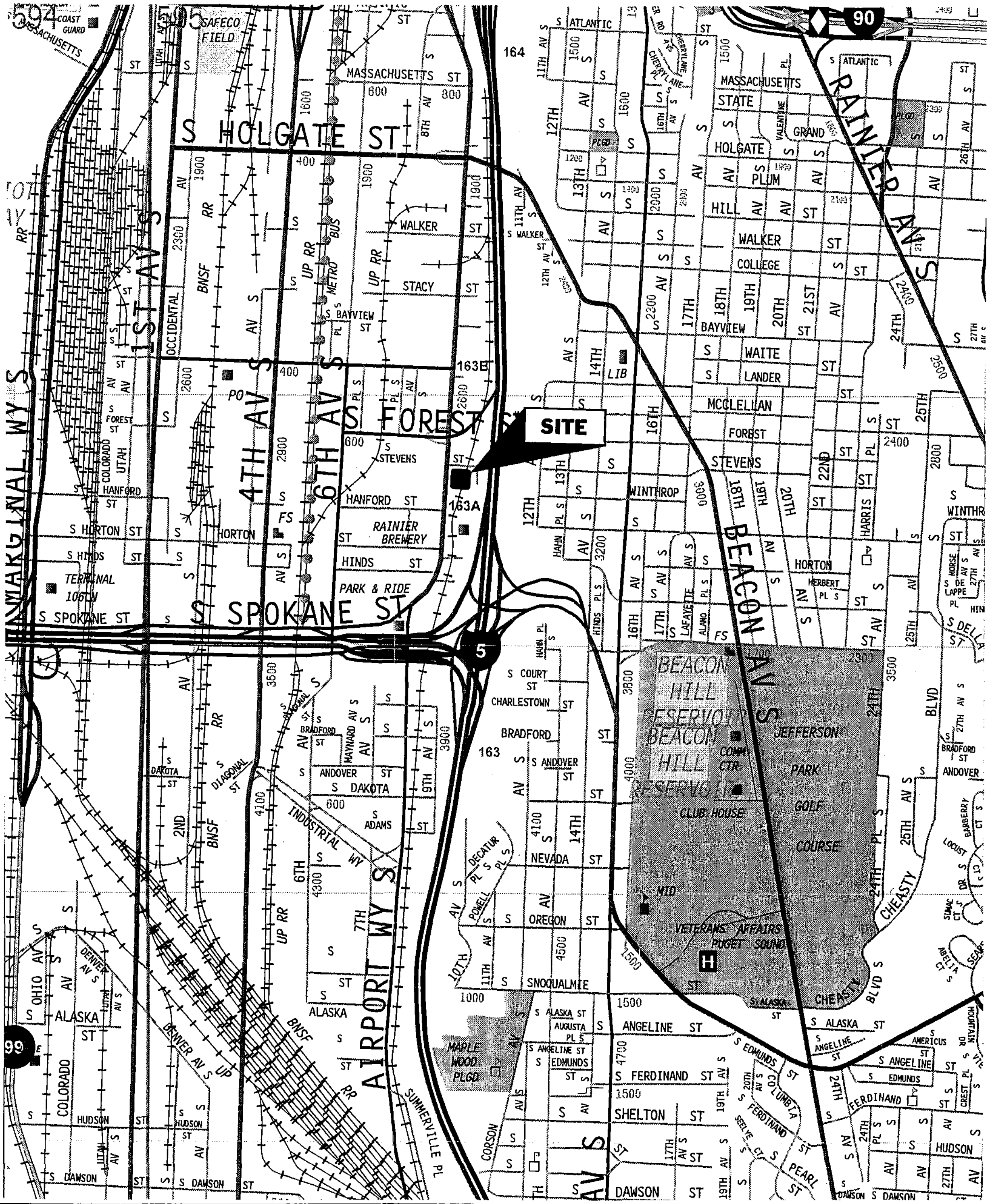
GEOTECH CONSULTANTS, INC.

D. ROBERT WARD
STATE OF WASHINGTON
29740
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 10/21/2007

4/9/07

D. Robert Ward, P.E.
Principal

DRW: jyb

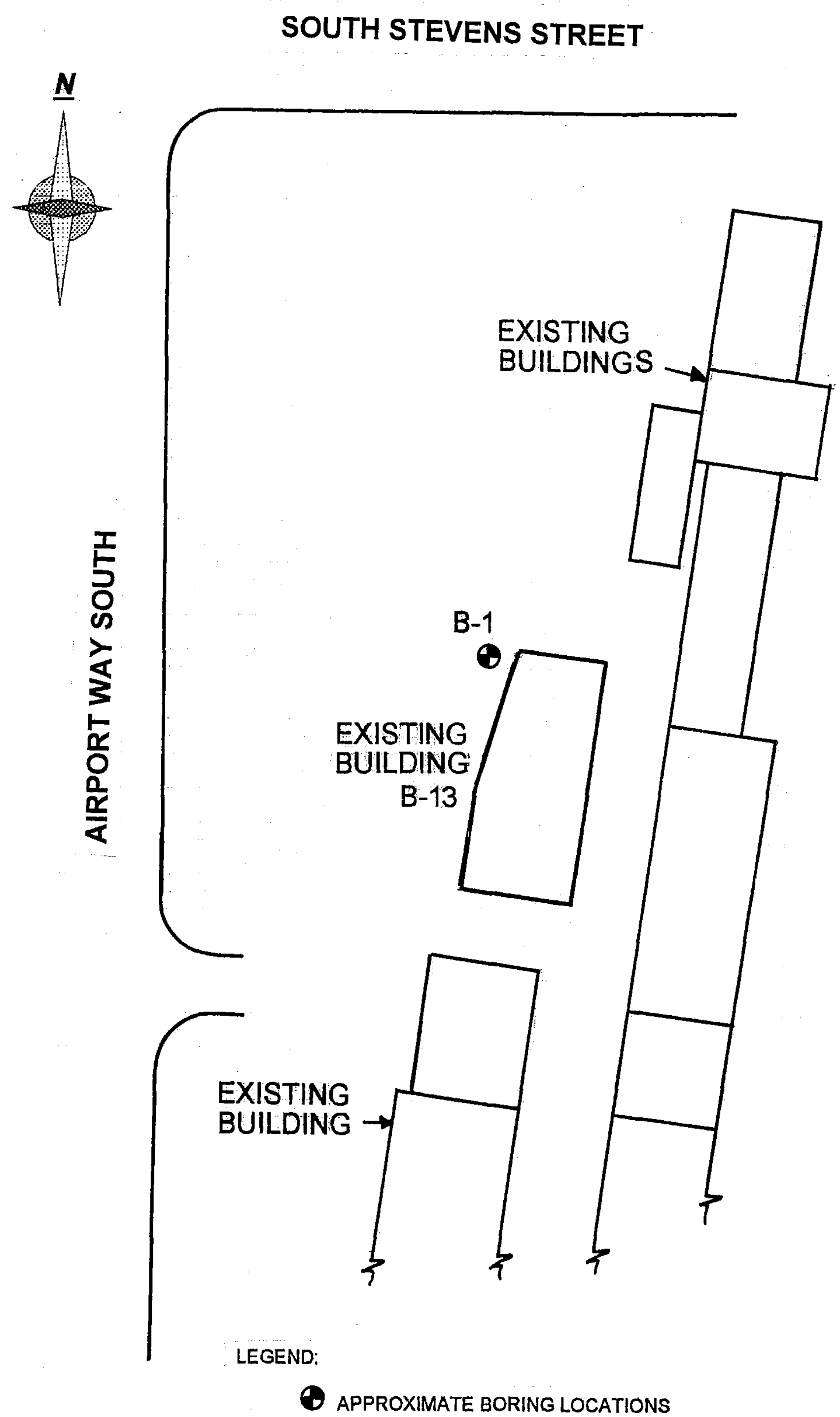


GEOTECH
CONSULTANTS, INC.

VICINITY MAP

31xx Airport Way South
Seattle, Washington

Job No:	Date:		Plate:
05402	Jan. 2006		1



GEOTECH CONSULTANTS, INC.

SITE EXPLORATION PLAN

31xx Airport Way South
Seattle, Washington

Job No:	Date:	Scale:	Plate:
05402	Jan. 2006	None	2

BORING 1

Depth (ft)	Well Design	Water Table	Blows per Foot	Sample	USCS	Description
			5	1	ML	Brown, sandy SILT with lenses of black ash, wet, loose
5			6	2		-becomes blue/gray, clayey, medium plasticity, soft
10			5	3	SP	Gray SAND, medium- to coarse-grained, wet, medium-dense
15		▼	15	4		
20			22	5	ML	Gray, clayey SILT, low to medium plasticity, very moist to wet, medium stiff to stiff
25			26	6		-becomes stiff
30			29	7		
35			29	8		
40						

*** Test boring was terminated at 39 feet due to auger refusal on November 2, 2005.**

*** Groundwater seepage was encountered at 19 feet during drilling.**

GEOTECH
CONSULTANTS, INC.

BORING LOG

31xx Airport Way South
Seattle, Washington

Job	Date:	Logged by:	Plate:
05402	Dec. 2005	DRW	3